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Basics of hydrology in urban areas

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BASICS OF HYDROLOGY IN URBAN AREAS

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TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

1. INTRODUCTION

Urbanization - the concentration of people in urban areas and the consequent expansion of these areas, is a characteristic of our time. Since 1950 the urban population of Canada has increased from slightly over 50 percent to about 75 percent of the total population. It is generally accepted that this trend for increased urban growth will continue through the remainder of the century.

In the past, land was plentiful and resources were abundant. Both were taken for granted, and their uncontrolled development was for many decades the accepted way of urban growth. Growth for the sake of growth became the goal, and cities competed openly with each other for industry and people. Indeed, if resources were considered, it was in terms of exploitation rather than environmental consideration.

The traditional storm drainage design philosophy was to collect the runoff and carry it away as rapidly as possible out of the boundaries of the urban watershed. To achieve this goal, all impervious areas such as streets and roofs were connected to a man-made network of gutters and conduits which has a considerably higher density and flow velocity than the natural drainage system. The design of the storm sewer system was made independently from studies related to flood control from rare events. Storm water was also considered to be clean and there was no concern with regard to pollution from separate sewer systems. Gradually, the negative aspects of this philosophy and the price of uncontrolled urban growth became apparent. The environment was rapidly becoming despoiled by a dramatic increase of peak flow at the outlet of the urban watersheds, increased incidence of local flooding, depletion of ground water and pollution of receiving streams.

The recognition of the problems associated with the traditional storm drainage design led to a significant increase in urban drainage investigations. Many new ideas have been suggested for the design and management of storm water drainage facilities. Instead of building ever larger and more expensive channels and conduits for carrying the increased runoff from urban areas, the present trend is for the retardation of flows, the attenuation of their peaks, and their diversion for treatment to abate pollution.

Whereas simple hydrologic methods were and still are adequate at providing design peak flows for conduit sizing, the implementation of the new urban drainage concepts requires the use of improved hydrologic tools. The design of a storage, for example, to reduce flow peaks is possible only through the use of hydrographs (Figure 1). The storage in an urban area is not necessarily confined to a reservoir but may be distributed over various elements of the watershed such as parking lots, rooftops, ponds or enlarged channels. Other methods of peak reduction are the retardation of flow by reduction of velocity or increase of infiltrated volumes.

In the past few years, a large number of models dealing with urban storm runoff, some of which include quality considerations, were developed. When considering hydrologic models for the design of a storm sewer system, a thorough understanding of the rainfall-runoff process is essential because various aspects of this process are modelled and require input parameters.

This paper will examine the general effects of urbanization on the hydrology of a watershed, give a qualitative description of the urban storm water runoff and, finally, describe some available methods and models for the quantitative determination of this runoff.

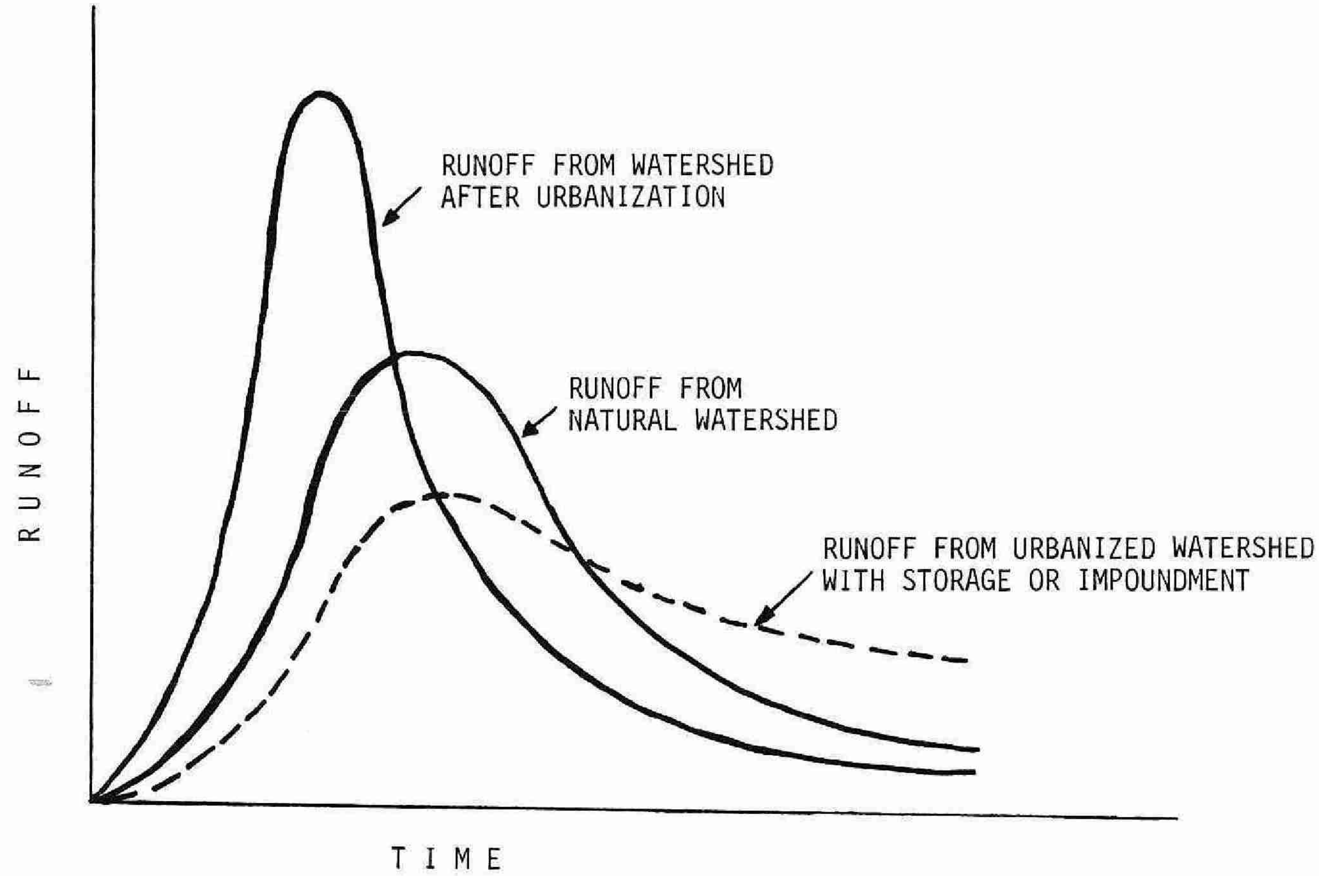


FIGURE I. EFFECT OF URBANIZATION AND STORAGE ON RUNOFF HYDROGRAPHS

2. THE HYDROLOGIC CYCLE

Water continuously circulates from one place to another and transforms from one state to another. This never-ending circulation and transformation of water is called the hydrologic cycle (Figure 2). The hydrologic cycle is subject to various complicated processes of precipitation, evaporation, transpiration, interception, detention, infiltration, percolation, storage and runoff.

It should be recognized that the hydrologic cycle has neither beginning nor end, as water evaporates from the land, oceans and other water surfaces to become part of the atmosphere. The moisture evaporated is lifted, carried and temporarily stored in the atmosphere until it finally precipitates and returns to the earth - either on land or oceans. The precipitated water may be intercepted or transpired by plants, may run off over the land surface to streams (surface runoff) or may infiltrate the ground. Much of the intercepted water and surface runoff is returned to the atmosphere by evaporation. The infiltrated water may be temporarily stored as soil moisture and evapotranspired; or percolate to deeper zones to be stored as groundwater which may be used by plants; or flow out as springs; or seep into streams as runoff; and finally evaporate into the atmosphere to complete the cycle (Gray, 1970).

2.1 THE HYDROLOGIC EFFECTS OF URBANIZATION

Urbanization through the human inhabitation and the consequent controlled development of a watershed represents man's intervention in the natural hydrologic cycle. Because urbanization is a gradual process, its hydrological impact is also gradual and is a function of the degree of urbanization. Savini and Kammerer (1961) analyzed the hydrologic effects associated with urbanization during a selected sequence of changes in land and water use. Their analyses are summarized in Table 1.

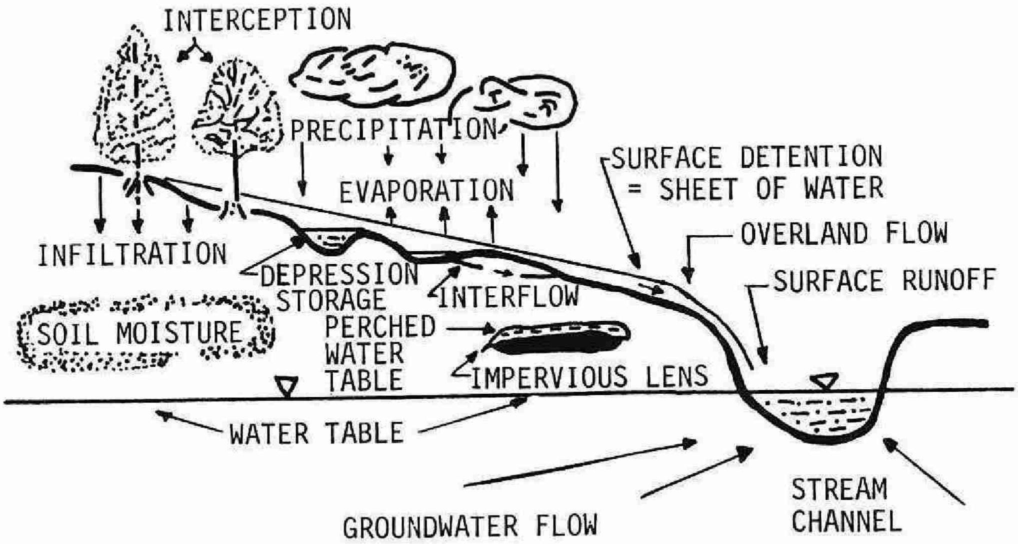


FIGURE 2. REPRESENTATION OF THE HYDROLOGIC CYCLE

Table 1. HYDROLOGIC EFFECTS DURING A SELECTED SEQUENCE OF CHANGES IN LAND AND WATER USE ASSOCIATED WITH URBANIZATION

Change in Land or Water Use	Possible Hydrologic Effect
A. Transition from pre-urban to early-urban stage:	
1. Removal of trees or vegetation. 2. Construction of scattered city-type houses and limited water and sewage facilities.	Decrease in transpiration and increase in streamflow. Increased sedimentation of streams.
3. Drilling of wells.	Some lowering of water table.
4. Construction of septic tanks and sanitary drains.	Some increase in soil moisture. Contamination of nearby wells or streams.
B. Transition from early-urban to middle-urban stage:	
1. Bulldozing of land for mass housing, some topsoil removed, farm ponds filled in.	Accelerated land erosion and stream sedimentation. Increased flood flows. Elimination of smallest streams.
2. Mass construction of houses, paving of streets, building of culverts.	Decreased infiltration, resulting in increased flood flows and lowered ground water levels. Occasional flooding at channel constrictions (culverts) on remaining small streams. Occasional overtopping or undermining of banks of artificial channels on small streams.
3. Discontinued use and abandonment of some shallow wells.	Rise in water table.

4. Diversion of nearby streams for public water supply.	Decrease in runoff between points of diversion and disposal.
5. Untreated or inadequately treated sewage discharged into streams or disposal wells.	Pollution of streams or wells. Death of fish and other aquatic life. Inferior quality of water available for supply and recreation at downstream populated areas.
C. Transition from middle-urban to late-urban stage:	
1. Urbanization of area completed by addition of more houses and streets and of public, commercial, and industrial buildings.	Reduced infiltration and lowered water table. Streets and gutters act as storm drains, creating higher flood peaks and lower baseflow of local streams.
2. Larger quantities of untreated waste discharge into local streams.	Increased pollution of streams and concurrent increased loss of aquatic life. Additional degradation of water available to downstream users.
3. Abandonment of remaining shallow wells.	Rise in water table.
4. Increase in population requires establishment of new water supply and distribution systems, construction of distant reservoirs diverting water from sources within or outside basin.	Increase in local streamflow if supply is from outside basin.
5. Channels of streams restricted at least in part to artificial channels and tunnels.	Increased flood damage (higher stage for a given flow). Changes in channel geometry and sediment load.

6.	Construction of sanitary drainage system and treatment plant for sewage.	Removing of additional water from the area, further reducing infiltration and recharge of aquifer.
7.	Improvement of storm drainage system.	A definite effect is alleviation or elimination of flooding of basements, streets, and yards, with consequent reduction in damages, particularly with respect to frequency of flooding.
8.	Drilling of deeper, large capacity industrial wells.	Lowered pressure in artesian aquifer; perhaps some local overdrafts (with-drawl from storage) and land subsidence. Overdraft of aquifer may result in salt-water intrusion in coastal areas and in pollution by brackish waters.
9.	Drilling of recharge wells.	Raising of water pressure surface.
10.	Waste water reclamation and utilization.	Recharge to ground water aquifers. More efficient use of water resources.

Quantitative evaluations of the effects of urbanization on the hydrology of a watershed are still in their initial stages as the techniques needed for such evaluations are still evolving. Most quantitative studies focused on the changes in the response of the watershed to precipitation input before and after urbanization.

In most of these studies, the percentage of built-up or impervious area in the watershed is used to characterize the degree of urbanization. Almost all such studies revealed the following effects of urbanization:

1. Urbanization causes an increase in the volume of stormwater runoff because of increased impervious cover;

2. Urbanization causes higher flood peaks because of the increased velocities of flow leaving roofs, gutters, lined channels or conduits; as opposed to natural surfaces, rivulets, or channels;

3. Urbanization causes a reduction in baseflow; and

4. Urbanization causes a decrease in the time lag of urbanizing watershed.

2.1.1 INCREASED RUNOFF VOLUMES

The increase in the volume of stormwater runoff due to urbanization can be demonstrated by the following example (Labadie and Grigg, 1976). A 25 mm rainstorm will produce a volume of precipitation of 1.64×10^6 m^3 over a city of 65 km^2. If the data given in Table 2 relative to percent imperviousness and runoff coefficient apply before and after urbanization, then the net increase in runoff quantity becomes evident.

Table 2. RUNOFF VOLUME INCREASE FROM URBANIZATION (Labadie and Grigg, 1976).

	Before Urbanization	After Urbanization
Area of Watershed	65 km^2	65 km^2
Percent Imperviousness	0%	40%
Runoff/Precipitation, %	15%	50%
Precipitation Volume	$1.64 \times 10^6 m^3$	$1.64 \times 10^6 m^3$
Runoff Volume	$0.25 \times 10^6 m^3$	$0.82 \times 10^6 m^3$
Aquifer Recharge	$1.39 \times 10^6 m^3$	$0.82 \times 10^6 m^3$
Increased Runoff	--------------------	+234%
Decreased Recharge	--------------------	-41%

The simple water balance given in Table 2 shows an increased runoff volume of 234% from the cited storm and a net loss to the aquifer of 41%. The size of the area for the above example would perhaps be equivalent to a city of 500,000 - 1,000,000 population. The changes due to an annual growth increment of, say, 3% would be small, but still significant. For example, one year's growth of 3% for the above city might add 1.94 km^2 to the urbanized area. The increase in runoff due to this added area from the storm cited would be 17,260 m^3. It is the continued acceptance of these increments of runoff that has caused the problems we face today.

2.1.2 INCREASED RUNOFF RATES

Reports of actual peak flow multipliers (or the ratio of peak discharge after urbanization to peak discharge before urbanization) vary over wide ranges. Some hypothetical calculations result in multipliers as high as five. A ratio such as this might hold for a small area, measured in acres. For larger areas, however, it is suspected that the ratios are smaller. A report from the U.S. Geological Survey presented the results of an empirical analysis which demonstrated multipliers ranging around the 2.0 mark, depending on values of percent imperviousness and storm frequency. Table 3 gives the results of this investigation (Stanlowski, 1974).

Table 3. RATIO OF PEAK DISCHARGE AFTER URBANIZATION TO PEAK DISCHARGE BEFORE URBANIZATION

Storm Recurrence Interval (years)	Index of man-made impervious cover (percent)				
	1	10	25	50	80
2	1.0	1.8	2.2	2.6	3.0
5	1.0	1.6	2.0	2.4	2.6
10	1.0	1.6	1.9	2.2	2.4
25	1.0	1.5	1.8	2.0	2.2
50	1.0	1.4	1.7	1.9	2.0
100	1.0	1.4	1.6	1.7	1.8

2.1.3 REDUCED BASEFLOW

An increase in the imperviousness of an urban area results in increased runoff volumes and decreased infiltration. As the amount of water available for soil moisture replenishment becomes smaller, the ground water recharge decreases and the baseflow diminishes. Thus, increased imperviousness has the effect of increasing flood volumes and peaks during storm periods and decreasing baseflow between storms. Klein (1979) investigated 27 small watersheds in Maryland to determine if a relationship exists between baseflow and the extent of watershed urbanization. Klein found a definite relationship between them as illustrated in Figure 3.

2.1.4 DECREASED TIME LAG

Many investigators also showed that, compared to rural watersheds, the average time lag for urban watersheds is smaller by 60 to 70 percent. However, the results from these studies cannot be easily compared because the investigators used different definitions for the time lag. Table 4 gives five different definitions for time lag and Figure 4 illustrates graphically these definitions. Figures 5 and 6 show comparisons between the average time lag as defined by Carter (1961) and Eagleson (1962) for natural and urban drainage basins and illustrate the reduction in time lag due to urbanization. In Figure 5 the average time lag is compared with the ratio $L/\sqrt{S_w}$, where L (in miles) is the length of the main stream and S_w is the weighted slope of the main stream expressed in feet per mile. In Figure 6 the variation of the average time lag is given against the ratio $LL_{ca}/\sqrt{S_w}$, where L_{ca} is the distance along the main stream from the basin outlet to a point opposite the center of gravity of the basin in miles.

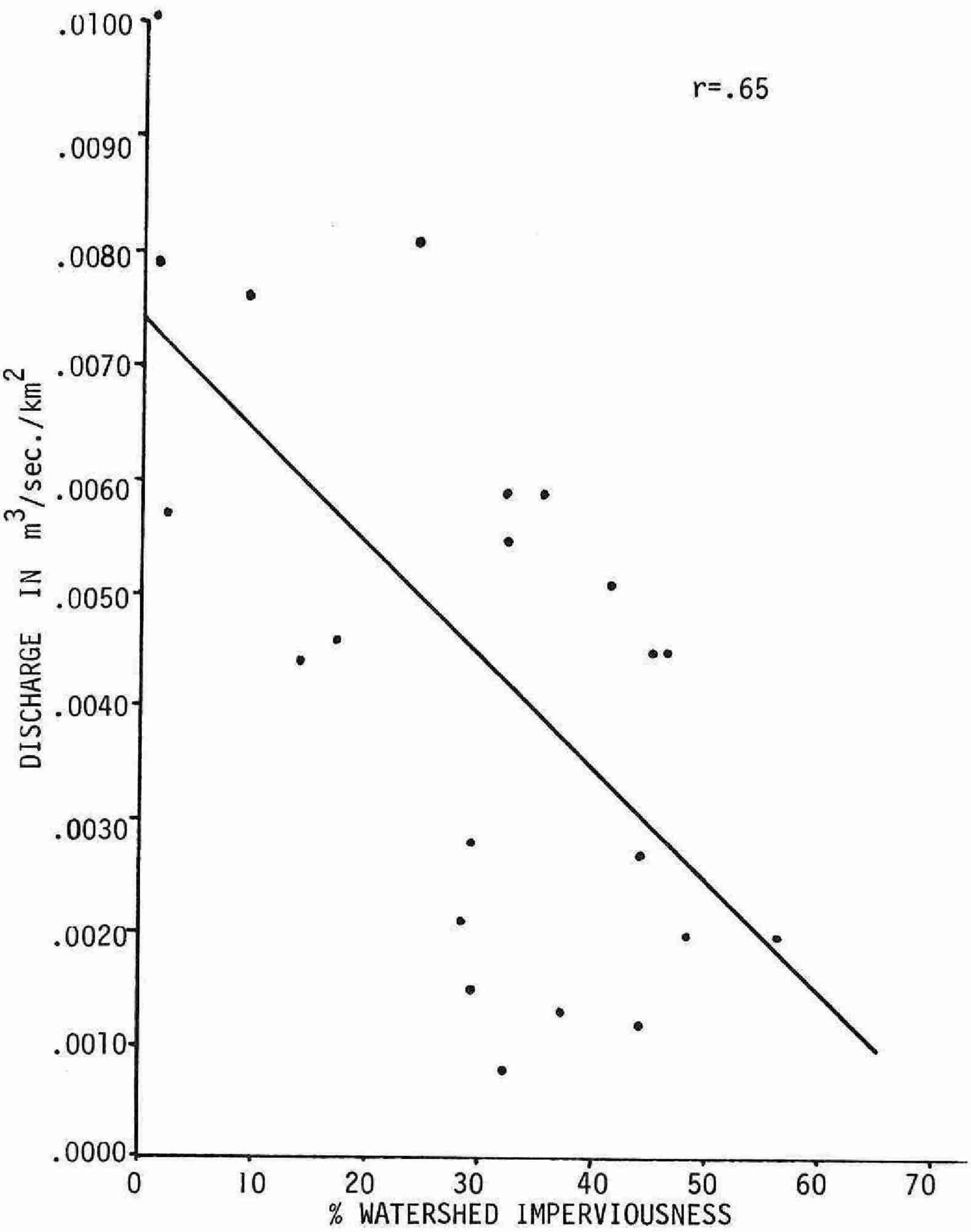


FIGURE 3. BASEFLOW VERSUS WATERSHED IMPERVIOUSNESS

Table 4. DEFINITIONS OF TIME LAG

Symbol	Definition	Reference
T_1	Time from centroid of excess rainfall to the peak of direct runoff hydrograph	Eagleson, 1962
T_2	Time from beginning of continuous excess rainfall to the peak of direct runoff hydrograph	Espey et al, 1966
T_3	Time from beginning of continuous excess rainfall to the centroid of direct runoff hydrograph	Linsley et al, 1958
T_4	Time from the centroid of excess rainfall to the centroid of direct runoff hydrograph	Carter, 1961
T_5	Time from the centroid of excess rainfall to the mid-volume of direct runoff.	Holtan and Overton, 1964

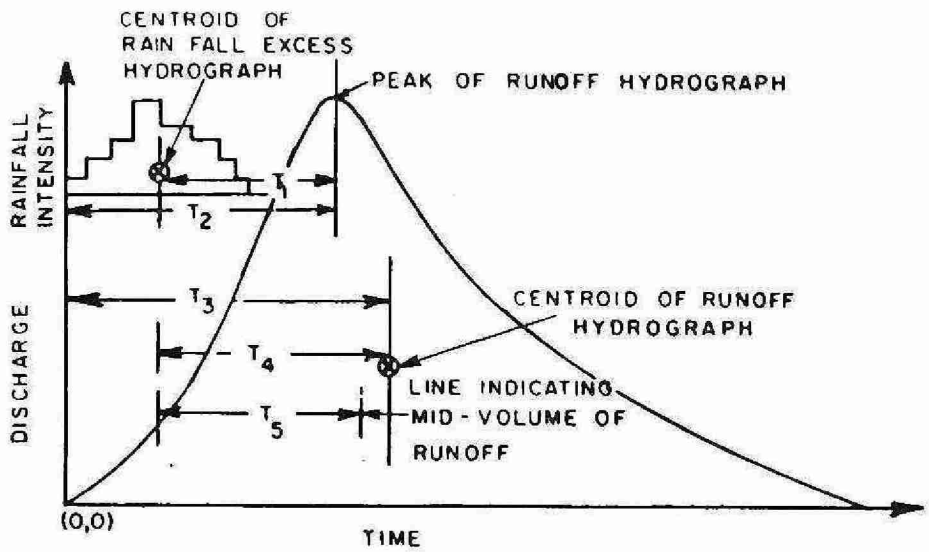


FIGURE 4. DEFINITIONS OF TIME LAG

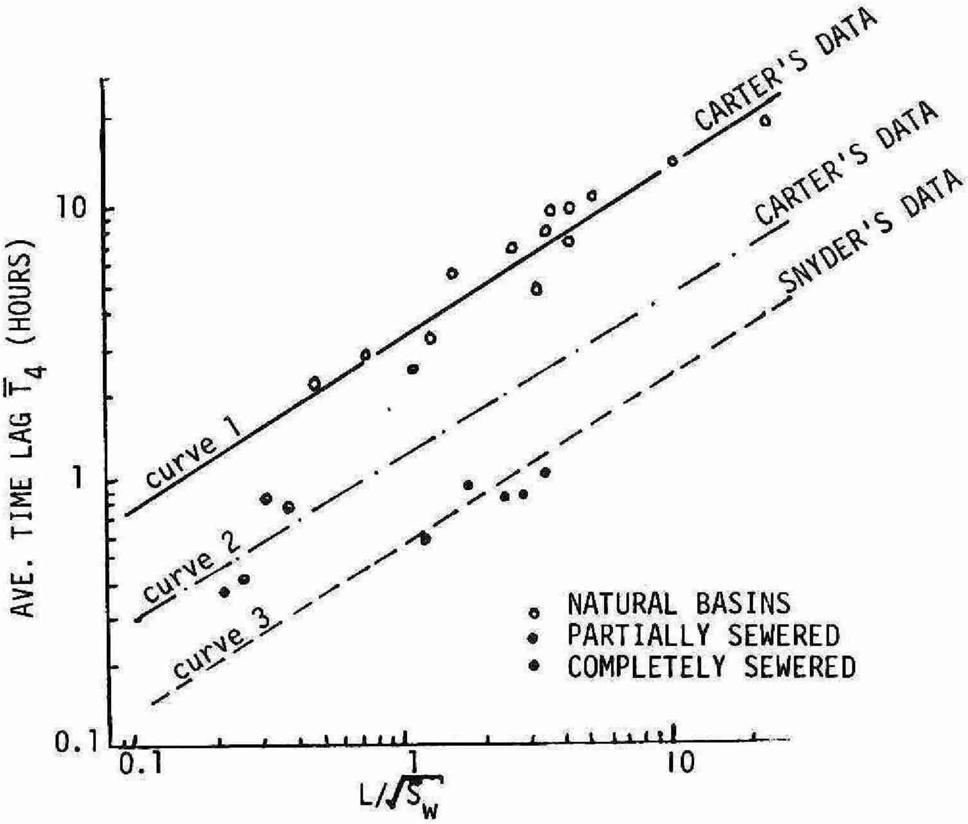


FIGURE 5. VARIATION OF AVERAGE TIME LAG WITH $L/\sqrt{S_w}$ (AFTER CARTER)

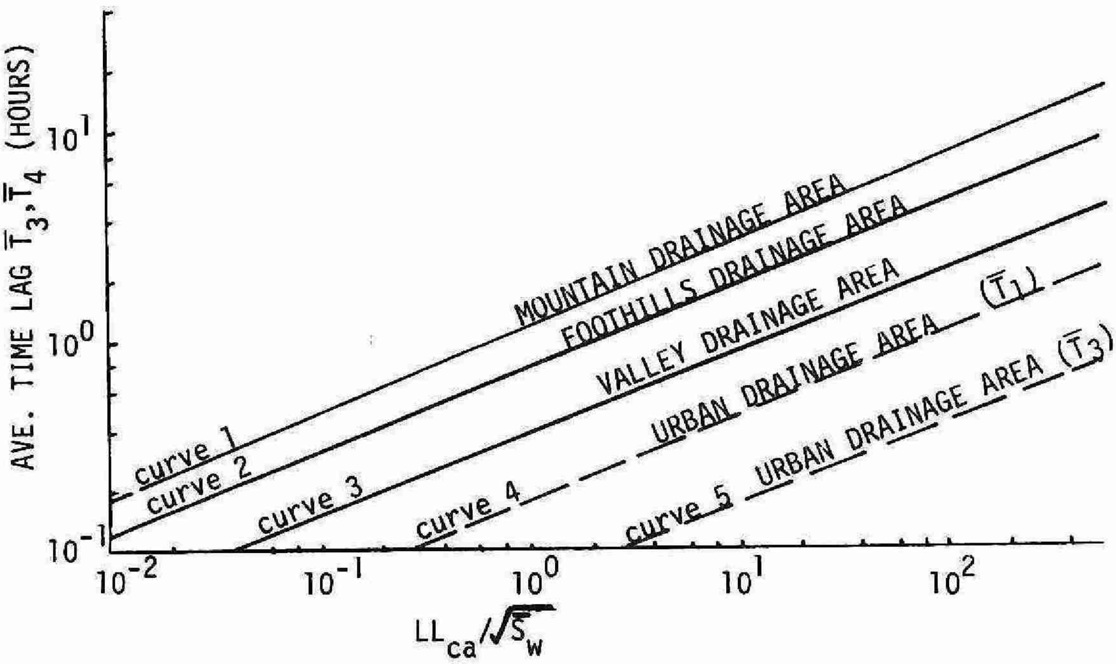


FIGURE 6. VARIATION OF AVERAGE TIME LAG WITH $LL_{ca}/\sqrt{S_w}$ (AFTER EAGLESON)

3. QUALITATIVE DESCRIPTION OF URBAN STORM WATER RUNOFF

3.1 GENERAL

Runoff is that part of precipitation which appears in streams. Depending on the source from which the flow is derived, runoff may consist of surface runoff, subsurface runoff, and ground water runoff. The surface runoff is that part of the runoff which travels over the ground surface and through channels to reach the basin outlet. The part of the surface runoff that flows over the land surface toward stream channels is called overland flow.

The subsurface runoff, also known as interflow, is the runoff due to that part of the precipitation which infiltrates the soil and moves laterally through the upper soil horizons towards the streams.

The ground water runoff, is that part of the runoff due to deep percolation of the infiltrated water which has passed into the ground, has become ground water, and has been discharged into the stream.

For the practical purpose of runoff analysis, total runoff in a stream channel is generally classified as direct runoff and baseflow. Direct runoff is that part of runoff which enters the stream promptly after the rainfall or snowmelt. It equals the sum of the surface runoff and the prompt subsurface runoff.

During a runoff-producing storm, the total precipitation may be considered to consist of precipitation excess and abstractions (Figure 7). The precipitation excess is that part of the total precipitation that contributes directly to the surface runoff. When the precipitation is rainfall, the precipitation excess is known as rainfall excess. The abstractions are the remaining parts which do not eventually become surface runoff, such as interception, evaporation, transpiration, depression storage, and infiltration.

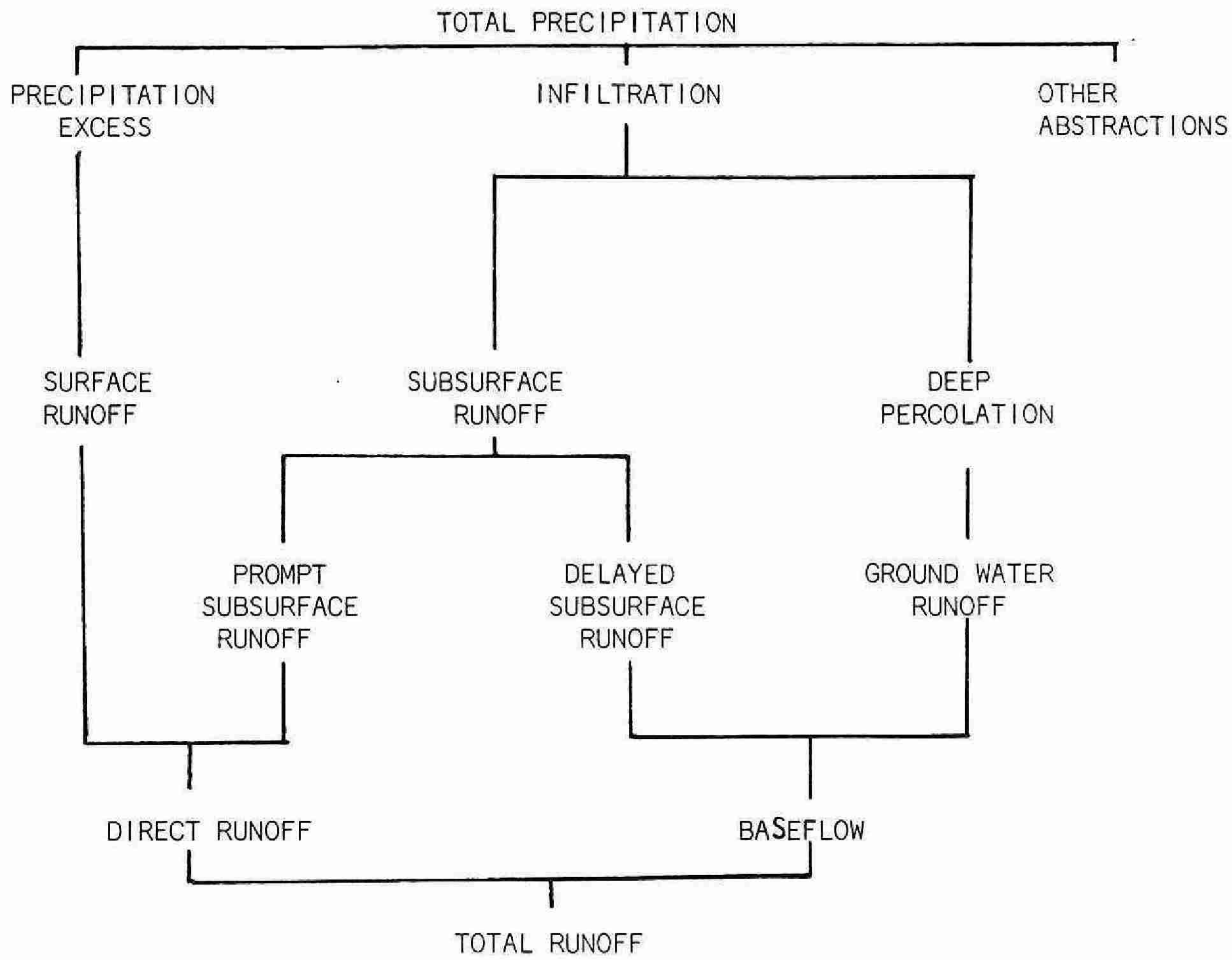


FIGURE 7. THE RAINFALL-RUNOFF PROCESS TERMINOLOGY

The part of precipitation that contributes entirely to the direct runoff may be called the effective precipitation or effective rainfall. It must be noted that in some procedures the subsurface runoff is entirely excluded from the direct runoff and then the effective precipitation (or rainfall) is equivalent to the precipitation (or rainfall) excess.

3.2 CHARACTERISTICS OF URBAN RUNOFF

3.2.1 COMPONENTS OF RAINFALL-RUNOFF PROCESS

The rainfall-runoff process within an urban watershed can be divided into two distinct components, one being hydrologic and the other hydraulic. The hydrologic component encompasses precipitation, infiltration, depression storage and overland flow. The hydraulic component deals with the summation of inlet flows and the routing of the resultant flows through the sewer network considering such factors as storage, friction losses and type of flow (gravity or pressure).

3.2.2 URBAN DRAINAGE SYSTEMS

Every urban watershed has two separate and distinct drainage systems. This is known as the "Dual Drainage Concept". One drainage system is called the minor system and consists of gutters, pipes, roof leaders, etc. The other is the major system and includes roads, stream channels, etc. The traditional urban drainage studies considered the minor system which was designed to permit the public the use of roads during the more frequent storms. The major system, on the other hand, is the route for runoff during times when the minor system is inoperable due to partial or total blockage, or when its capacity has been reached.

Typical minor systems are usually intended to have sufficient capacity to collect and transport the runoff from a storm that might be expected to occur once in a 2 to 10 year period. Storms that are expected to occur once in a 25 to 100 year period might be considered in the major system.

3.2.3 PERVIOUS-IMPERVIOUS AREAS

All areas in an urban watershed can be considered as either being pervious or impervious. The impervious areas do not allow water to infiltrate into the ground. These areas may or may not be directly connected to the sewer system. The degree of urbanization of a watershed is often defined by the percentage of imperviousness which is the ratio of the directly connected impervious areas to the total watershed area.

For the purpose of designing an urban drainage system, the following values of percent imperviousness are presented as a guide for areas of future development:

Land Use	% imperviousness
Single Family Residential	30 - 50
Semi-detached Residential	40 - 60
Apartments and Townhouses	50 - 70
Institutional	50 - 70
Commercial and Industrial	80 - 100
Parks and Open Spaces	0 - 10

3.2.4 INTERCEPTION

Quantitatively, rainfall interception by vegetation is rarely of importance in connection with urban storm drainage and may properly be ignored in design.

3.2.5 INFILTRATION

The flow of water through the soil is called infiltration. When rain falls on pervious areas in an urban watershed, some, if not all, will be abstracted by the infiltration of water into the soil. On the other hand, infiltration can be assumed to be zero for impervious areas.

The soil infiltration capacity depends on the soil type (especially its clay content), its texture and its structure. It is also affected by the vegetation cover, the slope of land and the antecedent precipitation period.

Generally, infiltration has a high initial rate that diminishes during continued rainfall towards a nearly constant lower rate. Many methods have been developed for evaluating infiltration. The majority of these methods reflect the characteristic exponential decrease in infiltration capacity from a maximum initial rate to a steady minimum rate.

Horton (1940) developed a mathematical equation for defining the rate curve of infiltration capacity:

$$I_f = f_o + (f_i - f_o)e^{-kt} \quad (1)$$

where I_f is the infiltration rate, f_i and f_o are the maximum and minimum infiltration rates, respectively, e is the base of natural logarithm, k is the decay rate of infiltration which depends primarily upon soil type, and t is the time in hours from the start of rainfall.

The values of infiltration parameters to be used in the above equation are difficult to assess without field measurements. Relative minimum infiltration capacities for three broad soil groups are as follows:

Soil group	f_o, mm/hour
Sandy, open-structured	10.0 - 25.0
Loam	2.5 - 10.0
Clay, dense-structured	0.25- 2.5

Information on soil types in Ontario are given in soil reports and maps which are published by the Ontario Ministry of Agriculture and Food.

3.2.6 DEPRESSION AND DETENTION STORAGES

Of the precipitation which reaches roofs, pavements, and pervious surfaces, some is trapped in the many shallow depressions of varying size and depth present on all surfaces. Any precipitation in excess of infiltration is held in these depressions until it eventually infiltrates or evaporates and does not, therefore, become part of the runoff. The amount of water required to fill these depressions is called the depression storage.

The term detention refers to the storage effect due to overland flow in transit. This temporary detention storage, together with the depression storage comprises the total surface storage.

The specific magnitude of depression storage has not been measured in the field because of obvious difficulties in obtaining meaningful data. The following numerical values can be used for estimating the surface storage:

Land Cover	Surface Storage (mm)
Large paved areas	0.5 - 3.5
Roofs - flat	2.5 - 7.5
Roofs - sloped	0.5 - 2.5
Lawn areas	5.0 -12.5
Open fields	5.0 -15.0

3.2.7 OVERLAND FLOW

Since the depressions in the urban watershed have different depths and filling rates, overland flow commences as soon as the infiltration capacity is satisfied. As was mentioned above, the depth of water detained in the overland flow forms the detention storage.

Overland flow is collected in gutters or ditches and subsequently enters the sewer system. The estimation of this overland supply by routing methods is theoretically possible for a plane surface. Prediction for real situations is complicated, however, because of the non-uniformity of the areas involved and the difficulty in estimating the parameters for the various losses. The routing of the inflows through the collecting system is a hydraulic problem which can be handled by proven methods. Several routing methods are available with different degrees of sophistication.

The flow in the storm sewer system is principally by gravity. Like natural drainage basins, smaller sewer branches unite with larger branches, and so on, until a main sewer is reached. The smallest catchment area, of the order of an acre in size, is that tributary to an inlet. For most smaller areas in the upper reaches of an urban drainage system, the time required to reach peak runoff after the beginning of a storm is a matter of minutes. Hence, high intensity, short duration rainfall is normally, the main, if not sole, type of precipitation contributing to critical runoff rates.

3.2.8 FLOW ROUTING IN SEWER NETWORKS

Flow routing may be defined as the procedure whereby the time and magnitude of the flood wave at a point in the sewer network is determined from the known or assumed data at one or more points upstream.

The hydrograph of the flow into an inlet is known as the inflow hydrograph and at the outlet it is known as the outflow hydrograph. Figure 8 represents typical inflow and outflow hydrographs. During the first portion of the flood wave, periods 0 to 8 on Figure 8, inflow exceeds outflow, and thus water is being stored in the sewer pipe. The area abdca, or the difference between the inflow and outflow hydrographs, represents the volume stored. During the time periods 9 to 17, outflow exceeds inflow, and thus water is being drawn from the storage. The volume of storage depleted is represented by area dfged, which equals abdca.

In general, the theoretical analysis of the movement of flood waves is quite complex, and the methods that attempt strict mathematical approach are generally impractical. In sewer networks, however, the analysis becomes extremely difficult due to extensive backwater effects, flow reversals, looped systems and sewer surcharging. Therefore, even with the assistance of high-speed computers, approximate numerical techniques and simplifying assumptions are necessary for the analysis of flow routing in sewer networks.

The fundamental law of unsteady flow in partially filled sewer pipes is based on the principles of conservation of energy and conservation of mass. It may be expressed by the following two partial differential equations:

$$\frac{\partial V_1}{\partial t} + V_1 \frac{\partial V_1}{\partial x} + g \cos\theta \frac{\partial h}{\partial x} = g\,(S_o - S_f) \tag{2}$$

$$\frac{\partial h}{\partial t} + D \frac{\partial V_1}{\partial x} + V_1 \frac{\partial h}{\partial x} = 0 \tag{3}$$

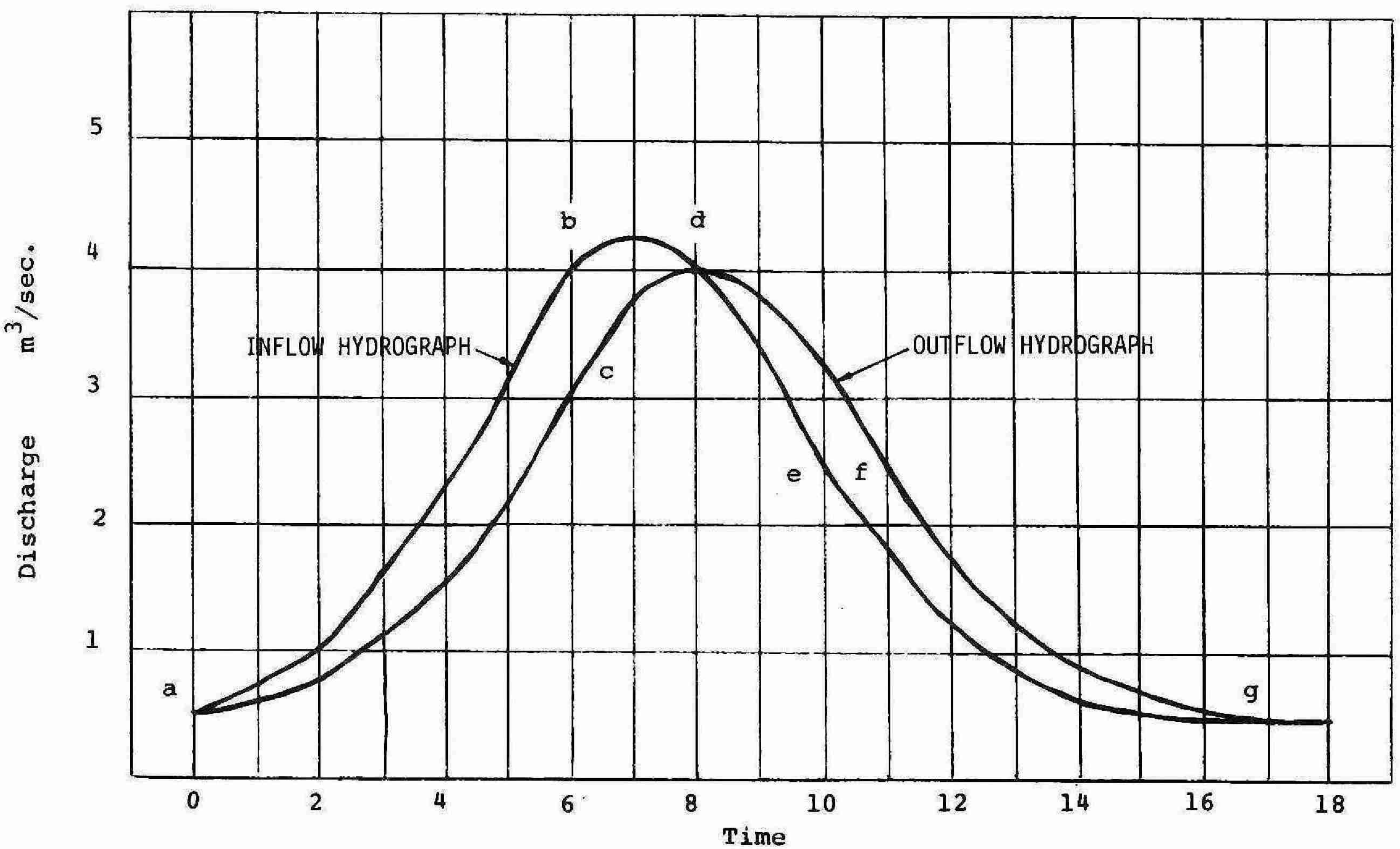


FIGURE 8. TYPICAL INFLOW AND OUTFLOW HYDROGRAPHS

where:

- x = longitudinal direction,
- V_1 = cross sectional average flow velocity along the x direction,
- t = time,
- g = gravitational acceleration,
- h = depth of flow measured perpendicular to x,
- θ = angle between the pipe bed and a horizontal plane,
- S_o = Sin θ = sewer slope,
- S_f = friction slope, and
- D = A/b = hydraulic depth of the flow where b is the width of the free surface.

Equations 2 and 3 are known as the St. Venant equations. They can be solved numerically provided that the initial conditions and the boundary conditions are specified. Neglecting the energy relationship represented by equation 2, many procedures of storage routing based on equation 3 have been developed. They have general applications and give relatively acceptable results.

The storage flow routing methods use a modified form of equation 3 which is written as:

$$I - O = \Delta S \tag{4}$$

or if expressed in finite time intervals:

$$1/2\ (I_1 + I_2)\ \Delta t - 1/2\ (O_1 + O_2)\ \Delta t = S_2 - S_1 \tag{5}$$

where the subscripts indicate the routing periods, and I, O, and S are instantaneous values of inflow, outflow, and storage, respectively, at the beginning of the routing periods indicated. Arranging equation 5 so that all known values are on the left, the expression becomes:

$$1/2\ (I_1 + I_2)\ \Delta t + S_1 - 1/2\ O_1\ \Delta t = S_2 + 1/2\ O_2\ \Delta t \tag{6}$$

Routing is accomplished by substituting the known values in the above equation to obtain $S_2 + 1/2\ O_2\ \Delta t$.

Then O_2 is obtained from the relationship between O_2 and S_2 + 1/2 $O_2 \Delta t$ (Chow, 1964).

Other methods, based on time displacement of average inflow values, have been used successfully in open-channel flow routing. The applicability of these methods to sewer networks flow routing is limited.

By considering both St. Venant equations, more theoretically acceptable methods are possible, but they suffer the handicap of greater complexity.

4. QUANTITATIVE DETERMINATION OF URBAN STORM WATER RUNOFF

Urban drainage design practices are currently undergoing a transition period. Some of the new methods being used or proposed have not yet been entirely proven by practical application since the transition to new techniques is a long-term continual process.

Precipitation and its distribution in time and space have to be characterized for runoff computations. The methods of such characterization depend largely upon the methods of analysis to be employed. Basic methods of rainfall and runoff analyses are presented in the following sub-sections.

4.1 RAINFALL ANALYSIS

4.1.1 DESIGN FREQUENCY

The design of storm drainage facilities is related to the degree of protection required which is usually expressed in terms of the storm recurrence interval in years. The primary objective of the frequency analysis of precipitation data is to determine the recurrence interval of a storm event of a given magnitude (x). The average interval of time within which the magnitude (y) of the event will be equalled or exceeded once is known as recurrence interval, return period, or simple frequency.

If a storm event equal to or greater than x occurs once in T years, the probability P $(X \geq x)$ is equal to 1 in T cases or:

$$P(X \geq x) = 1/T \tag{7}$$

and

$$T = 1/P(X \geq x) \tag{8}$$

where

T = return period, and

$P(X \geq x)$ = probability of occurrence that the event will be equalled or exceeded.

In engineering calculations of runoff, it is usually assumed that the frequency of occurrence of rainfall event is identical to the frequency of occurrence of the resulting runoff. Such an assumption is not correct because the runoff resulting from a rainfall with a particular return frequency is a function of the antecedent soil conditions, the spatial variation of rainfall over the drainage area and the shape of the drainage area.

The selection of the proper design frequency for drainage projects is a compromise between periodic inconveniences and damages due to flooding and the cost of preventing this flooding. Normally, the minor drainage system is designed for relatively frequent storms (recurrence 2-10 years). On the other hand, the major drainage system is designed for less frequent storms (recurrence 25-100 years).

4.1.2 RAINFALL INTENSITY - DURATION - FREQUENCY CURVES

Storm and combined sewers are usually designed to flow full without surcharging at some selected frequency, such as on an average of once in five years or once in ten years. The Rational Method is the most popular in the design of sewers. The only type of rainfall information needed for use of the Rational Method is an array of intensities for each of several durations from which various frequencies can be interpolated. The result is called "intensity-duration-frequency curves".

The rainfall intensity-duration-frequency curves are based on historical precipitation records. Such curves have been derived for a large number of municipalities in Canada. Bruce (1968) has developed "Rainfall-Intensity-Duration-Frequency Maps for Canada" using available rain gauge data.

In principle, one would search the rainfall record for each storm at a given station for the largest value over a particular duration. Experience has shown that many of the original data have practically no significance in the analysis because the hydrologic design is usually governed by a few critical conditions only. Thus, the complete duration series of rainfall data is not always used. In order to save labour and time, the data of insignificant magnitude are excluded. For this purpose, two types of data are generally selected from the complete-duration series: the partial-duration series and the extreme-duration series.

The partial-duration series or partial series, is a series of data which are so selected that their magnitude is greater than a base value.

The extreme-value series include the largest or the smallest values with each value selected from an equal time interval in the record. For annual largest values, the series is called an annual maximum series.

From the logical point of view, the selection of hydrologic data in designing a structure may be judged by the type of structure or project. The partial-duration series should be used if the second largest values in the year would affect the design. For practical purposes, the partial-duration series and the annual maximum series do not differ much except in the values of low magnitude. It is possible to convert from one series to another using the conversion factors given in Table 5.

Table 5. FACTORS FOR CONVERTING ANNUAL SERIES TO PARTIAL-DURATION SERIES

2 - year return period	1.14
5 - year return period	1.04
10 - year return period	1.01
Return periods for longer than 10 years	1.00

In deriving intensity-duration-frequency relations, rainfall values for each duration are regarded independently from other durations, the first step being a separate ranking of values for each duration in descending order of size. A mathematical fit is made to the array of intensities for each duration using the Gumbel analysis or the Log-Pearson Type III analysis.

The Gumbel analysis computes the rainfall (X) with a specified duration and return period according to the following equation:

$$X = \overline{X} + KS \qquad (9)$$

where $\overline{X}$ and S are the mean and standard deviations of the annual maximum series and K is the Gumbel's frequency factor which is a function of the number of data in the series and the required return period. The Gumbel Method is generally accepted for extreme value analysis of rainfall events and is used by the Atmospheric Environment Service.

The following similar equation is used for the Log-Pearson Type III analysis:

$$\text{Log } X = \overline{X}' + K'S' \qquad (10)$$

where $\overline{X}'$ and S' are the mean and standard deviation of the logarithm of the extreme value series for a specified duration. Whereas, K' is the Pearson Type III frequency factor which is a function of the return period and the coefficient of skewness of the data set.

In the last step, fitted values for each duration for each mean recurrence interval are plotted with intensity as ordinate and duration as abscissa; and smooth-fitted lines are drawn through points of equal frequency (Figure 9).

The rainfall intensity-duration-frequency curves offer an adequate rainfall characterization for the Rational Method. In the Rational Method, the rainfall intensity is assumed to be constant for a selected storm duration which equals the time of concentration of the catchment under consideration. The assumption of a constant rainfall intensity is not acceptable in some other methods of runoff computation which require complete design storm hyetographs.

4.1.3 DESIGN STORMS

Design storm hyetographs, which can be defined as graphs showing rainfall intensities over specified areas with respect to time, are a fundamental requirement for most hydrologic simulation models.

The design storm hyetograph is usually obtained in one of the following two ways:

a) It is derived from the intensity-duration-frequency curve by the method developed for the City of Chicago by Keifer and Chu (1957) and subsequently modified by others;

b) It is produced by averaging the time intervals of historical storms by using the concept of average mass curves as employed by the Illinois State Water Survey.

The design storm hyetographs for various frequencies can be derived from the intensity-duration-frequency curves using the methods developed by Bandyopadhyay (1972) and by Keifer and Chu (1957).

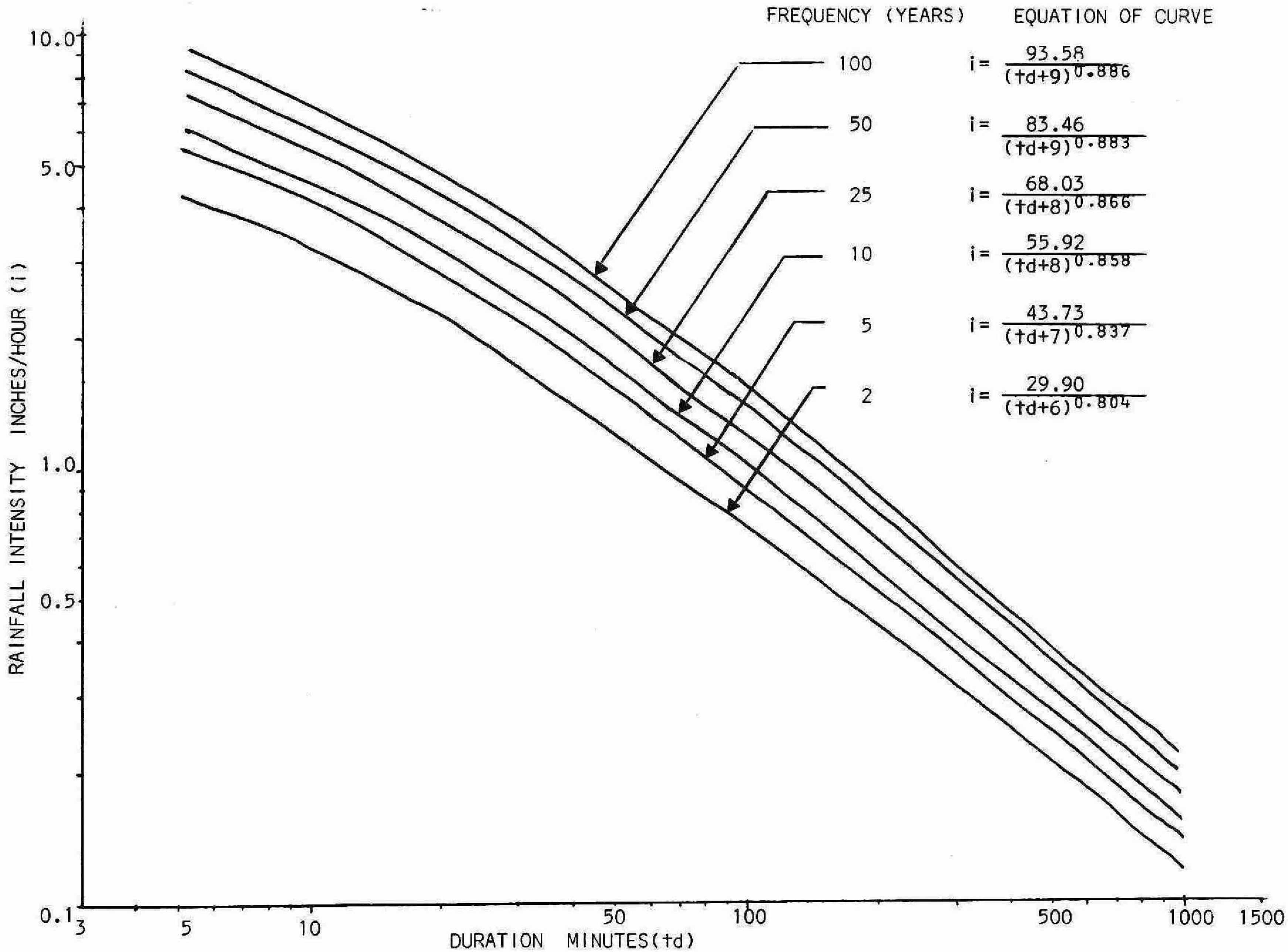


FIGURE 9. INTENSITY-DURATION-FREQUENCY CURVES (BASED ON 12 YEARS OF HAMILTON ROYAL BOTANICAL GARDEN RAIN GAUGE DATA, DILLON, 1977)

The first step is to obtain an equation for the rainfall intensity duration curves in the form of:

$$i = \frac{a}{(t_d + c)^b} \tag{11}$$

where i is the intensity, t_d is the duration and a, b and c are constants. The constants are obtained for each return period by fitting a regression equation, equation 12, to the rainfall intensity and duration data.

$$\text{Log } i = \text{Log } a - b \text{ Log } (t_d + c) \tag{12}$$

The next step is to derive the value of 'r' which represents the ratio of the time before the peak occurs to the storm duration time. This value of 'r' can only be derived from the existing rainfall records. The method involves calculating the mean values of mass antecedent rainfall and the mean location of the peaks for various rainfall durations for a series of rainfall events.

Once the values of a, b, c and r have been determined, the hyetograph can be plotted using the following equations:

i) Before the Peak:

$$i_b = \frac{a\left[(1-b)\frac{t_b}{r} + c\right]}{\left(\frac{t_b}{r} + c\right)^{1+b}} \tag{13}$$

ii) After the Peak:

$$i_a = \frac{a\left[(1-b)\frac{t_a}{1-r} + c\right]}{\left(\frac{t_a}{1-r} + c\right)^{1+b}} \tag{14}$$

where t_a is the time after the peak intensity and t_b is the time before the peak intensity (Figure 10).

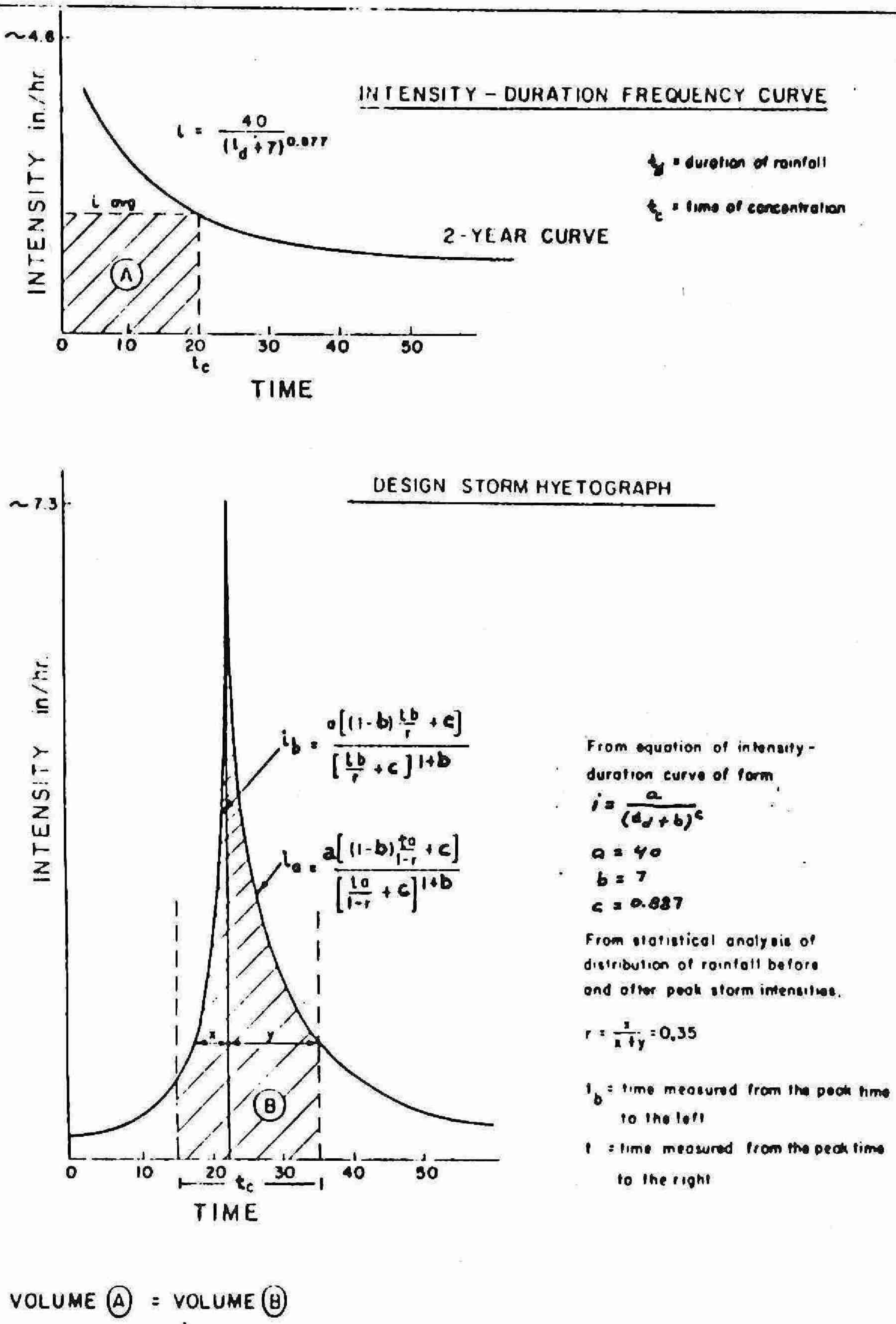


FIGURE 10. DERIVATION OF 2 YEAR DESIGN STORM FROM INTENSITY-DURATION FREQUENCY CURVE

For design purposes, the above defined hyetograph curve will generally have to be discretized. For example, a time step of 5 minutes can be assumed to be reasonable for discretization purposes, since the design hyetograph curves were originally derived from discrete rainfall data at minimum time intervals of 5 minutes.

The recommended method of discretizing the design hyetograph curves for the various return periods is as follows:

i) Select the time step Δt (e.g. 5 minutes).

ii) Compute the peak rainfall intensity from the following equation:

$$i_p = \frac{a}{(\Delta t + c)^b} \tag{15}$$

iii) Distribute the time interval selected (Δt) around the peak as r Δt before the peak and (1-r) Δt after the peak.

iv) Compute additional points before and after the peak until the derived intensity ordinates are insignificant.

The general integral form of the hyetograph curve before the peak is given by:

$$\int_{t_{b_1}}^{t_{b_2}} i_b \, dt_b = \left[\frac{a \, t_b}{(c + \frac{t_b}{r})^b} \right]_{t_{b_1}}^{t_{b_2}} \tag{16}$$

The general integral form of the hyetograph curve after the peak is given by:

$$\int_{t_{a_1}}^{t_{a_2}} i_a \, dt_a = \left[\frac{a\, t_a}{\left(c + \frac{t_a}{1-r}\right)^b} \right]_{t_{a_1}}^{t_{a_2}} \tag{17}$$

Figure 11 illustrates the 2-year discretized design storm hyetograph for the City of Burlington (Dillon, 1977).

An alternative method of discretizing design storm hyetographs is as follows:

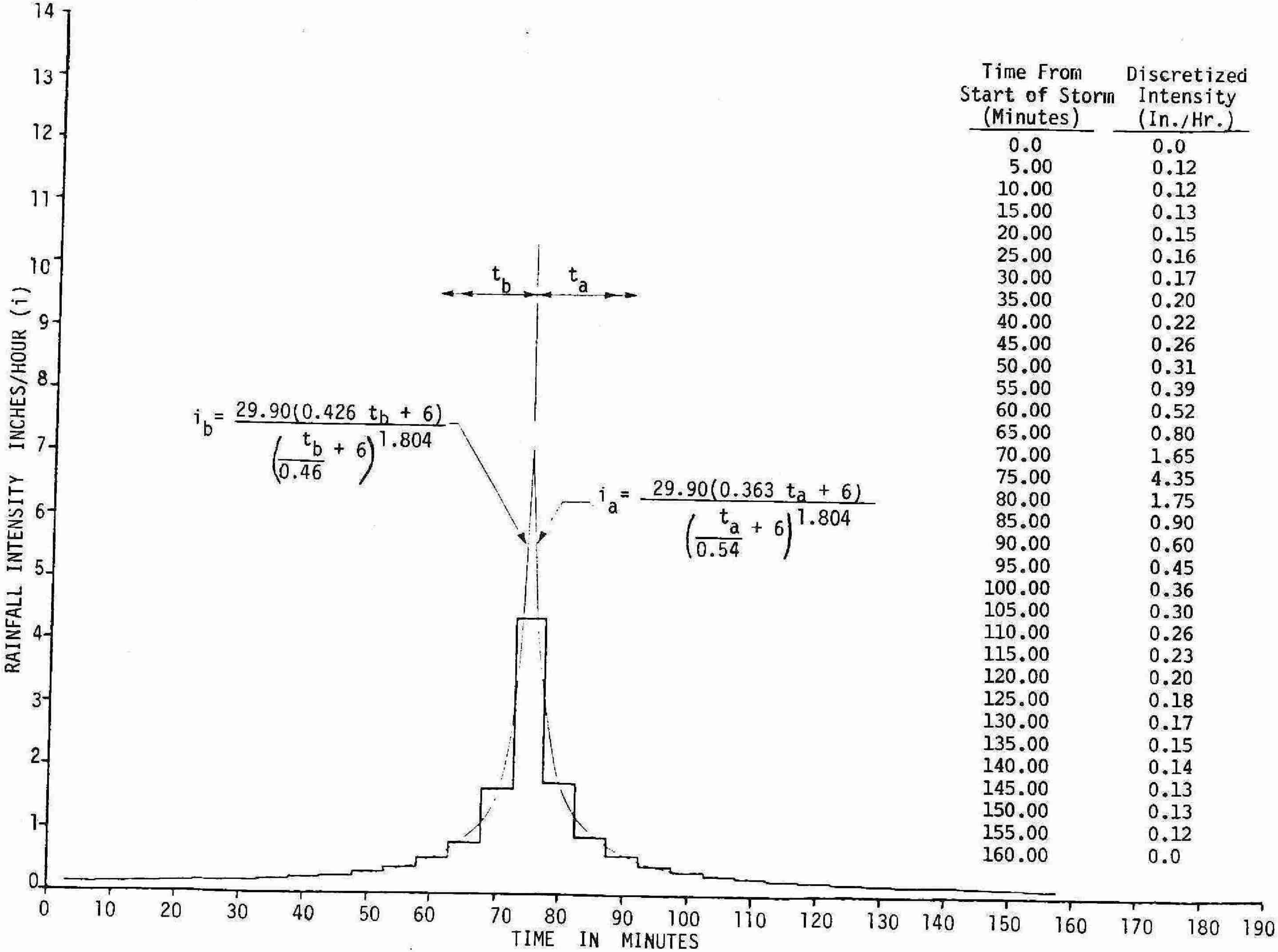
i) Produce the rainfall mass curve for the frequency under consideration.

ii) Locate the peak (point of contraflexure of mass curve).

iii) Distribute the time interval selected (Δt) around the peak as $r\,\Delta t$ before the peak and $(1-r)\ \Delta t$ after the peak.

iv) Derive the discrete ordinates as indicated in Figure 12.

Difficulties with the derivation of synthetic design storms, as well as uncertainties involved in these storms, led to the development of an alternative approach - adoption of a historical design storm. The selection of such a storm is done either directly or indirectly.



Time From Start of Storm (Minutes)	Discretized Intensity (In./Hr.)
0.0	0.0
5.00	0.12
10.00	0.12
15.00	0.13
20.00	0.15
25.00	0.16
30.00	0.17
35.00	0.20
40.00	0.22
45.00	0.26
50.00	0.31
55.00	0.39
60.00	0.52
65.00	0.80
70.00	1.65
75.00	4.35
80.00	1.75
85.00	0.90
90.00	0.60
95.00	0.45
100.00	0.36
105.00	0.30
110.00	0.26
115.00	0.23
120.00	0.20
125.00	0.18
130.00	0.17
135.00	0.15
140.00	0.14
145.00	0.13
150.00	0.13
155.00	0.12
160.00	0.0

FIGURE 11. 2 YEAR DISCRETIZED DESIGN STORM HYETOGRAPH (DILLON, 1977)

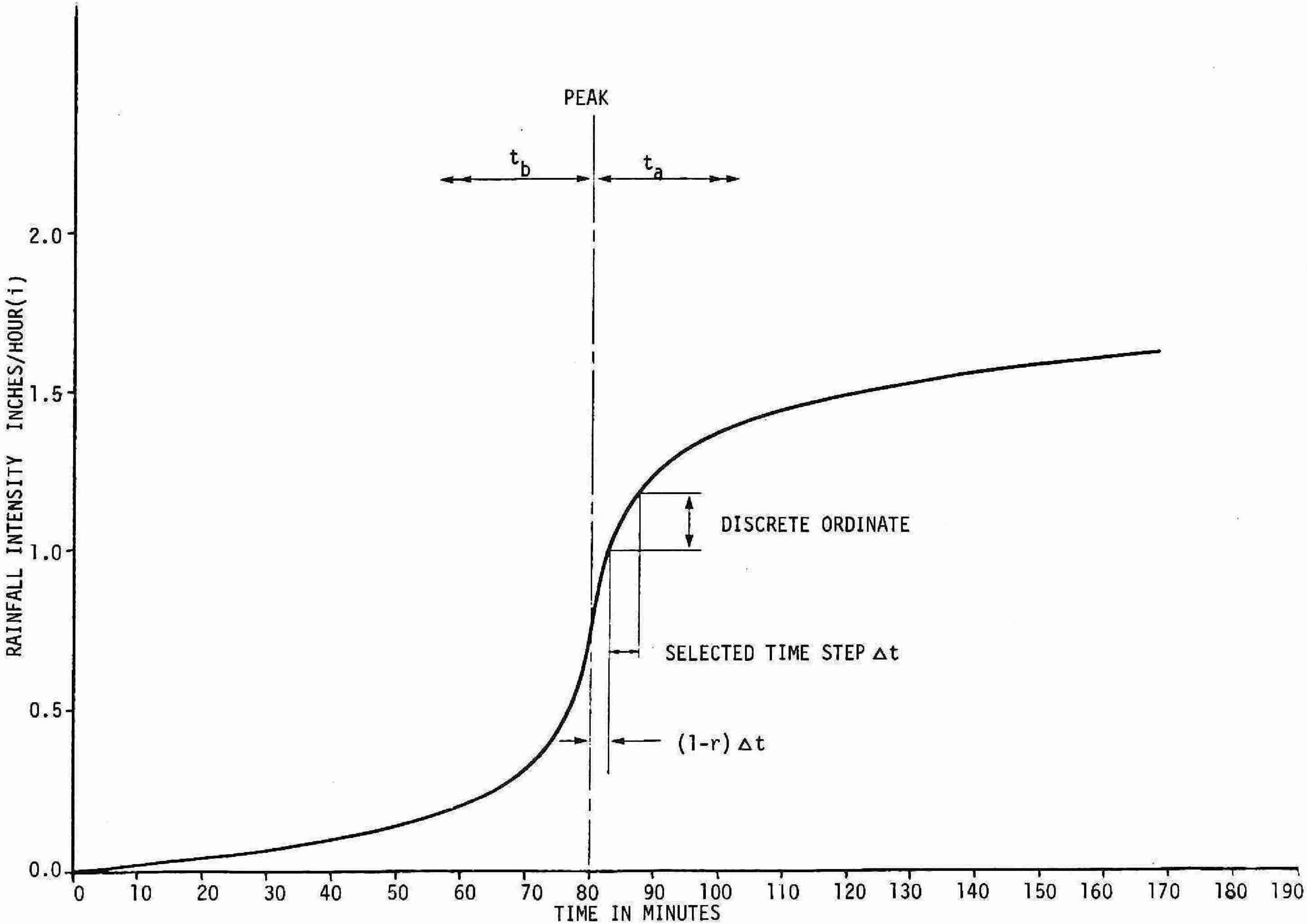


FIGURE 12. DESIGN STORM DISCRETIZATION USING THE RAINFALL MASS CURVE

Direct method - a historical storm whose characteristics are well documented is selected for future design. The frequency of occurrence of these historical events can be estimated. The approach described may be applicable to major drainage elements. Minor drainage elements are typically designed for more frequent events.

Indirect method is based on the frequency of occurrence of runoff events. A precipitation record is first translated by means of a simple continuous simulation model into a runoff record. A statistical analysis of the runoff record is then performed and the frequency of occurrence of various runoff flows is determined. For a selected runoff flow the corresponding storm can be identified and used as a historical design storm on catchments of similar size in the studied area.

4.2 RUNOFF ANALYSIS

Quantitative estimates of urban runoff can be made using one of the following four basic techniques, or a combination thereof:

Empirical formulae, unit hydrograph techniques, statistical regression techniques, and hydrologic simulation models.

4.2.1 EMPIRICAL FORMULAE, THE RATIONAL METHOD

A large number of empirical formulae for relating runoff to rainfall have been developed over the years. The most popular of these formulae is the Rational Method which was first introduced in 1889. It is currently widely used in the design of minor drainage systems. The method, however, is restricted to the computation of peak runoff.

In the Rational Method, runoff is related to rainfall intensity by the formula:

$$Q = CiA \tag{18}$$

where:

Q = the peak runoff rate in cfs,

C = the runoff coefficient which represents the drainage area characteristics,

i = the average rainfall intensity in in/hr for a duration equal to the time of concentration for a selected rainfall frequency, and

A = the size of the drainage area in acres.

The basic assumptions made when the Rational Method is applied are:

i) The peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point;

ii) The frequency of the peak discharge is the same as the frequency of the average rainfall intensity;

iii) The time of concentration is the time required for the runoff to become established and flow from the most remote part of the drainage area to the point under consideration.

The drainage area A, contributing to the point under consideration, is the only parameter in the Rational Method which can be determined precisely. The areas are usually delineated on suitable maps and then measured for area.

The determination of the runoff coefficient C is the most difficult. This is because one must take into account the many variables in the rainfall-runoff process such as infiltration, ground slope, ground cover, surface storage (detention and depression), shape of drainage area, antecedent precipitation and soil conditions, etc.

Guidance for selection of coefficient C is provided by Table 6 which shows commonly used values in accordance with the type of development and local soil characteristics. The recommended runoff coefficients in Table 6 should only be used for storm frequencies up to 10 years.

In order to account for antecedent precipitation conditions for the less frequent, higher intensity storms, the derived runoff coefficients should be multiplied by 1.1, 1.2 and 1.25 for storm frequencies of 25, 50 and 100 years respectively, the product, however, should not exceed 1.0.

The first step in determining the average rainfall intensity (i) is to derive the time of concentration. This is achieved by summing up the inlet time (i.e. the time for runoff to flow across the surface to the nearest inlet) and the time of flow in the sewer pipe to the point under consideration. The latter time can readily be estimated from the properties of the sewer. The inlet time, however, will vary with the distance of surface flow, surface slope, surface storage, surface cover, antecedent rainfall and the infiltration capacity of the soil. Figure 13 is presented to aid in the estimate of inlet times.

Once having established the time of concentration, the average rainfall intensity (i) for a rainfall duration (t_d) equal to the time of concentration can be derived for a selected rainfall frequency from the rainfall intensity-duration-frequency graph.

Table 6. RUNOFF COEFFICIENTS

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighbourhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Character of Surface	Runoff Coefficients
Pavement	
Asphalt or Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	
Lawns, sandy soil	
Flat 2%	0.05 to 0.10
Average, 2 to 7%	0.10 to 0.15
Steep, 7% or more	0.15 to 0.20
Lawns, heavy soil	
Flat, 2%	0.13 to 0.17
Average, 2 to 7%	0.18 to 0.22
Steep, 7% or more	0.25 to 0.35

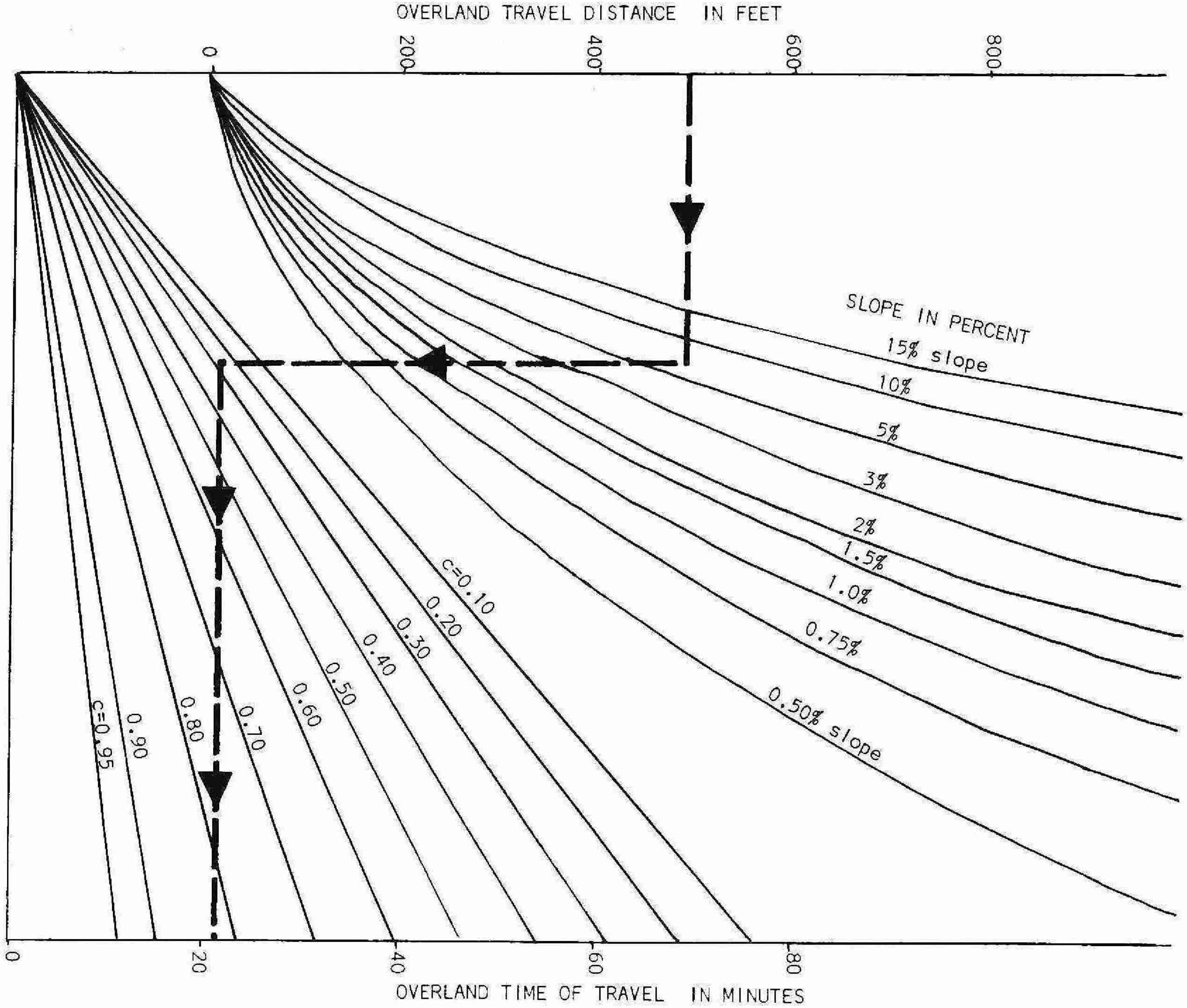


FIGURE 13. GRAPH OF OVERLAND FLOW TIME

It is important to note that the average intensity values thus derived have no time sequence relation to the actual rainfall pattern during the storm, since the intensity-duration-frequency curves are not a time sequence curve of precipitation.

Limitations of the Rational Method

When using the Rational Method for design purposes, the limitations of the method must be recognized. The more important of these limitations are:

i) The Rational Method does not account for the temporal and spatial variation of the rate of rainfall, nor does it account for the variation of the contributing area and the rate of contribution.

ii) The lumping of the physical factors affecting runoff into a single runoff coefficient C involves large uncertainties and precludes any possibilities of modifying individual factors. Furthermore, the runoff coefficient is not constant with time as is usually assumed, when applying the method.

iii) The Rational Method only estimates the peak runoff and not the actual storm runoff hydrograph, which is one of the requirements for the design of storage facilities.

iv) The estimated time of concentration especially for flat areas can be very inaccurate and consequently also the rainfall intensity for that time of concentration.

v) The Rational Method does not allow for retardation by storage and momentum of flow in channels.

Despite the aforementioned limitations, the Rational Method is currently widely used and it is well accepted by the engineering profession. It must be recognized, however, that the method is most suited to the design of drainage elements in small watersheds.

4.2.2 UNIT HYDROGRAPH

Since the 1930's attempts have been made to develop alternative improved methods for flow predictions based on hydrologic parameters.

In 1932 Sherman developed the unit hydrograph theory which became a universally accepted method of determining the flow hydrograph from rainfall. Sherman postulated that the runoff hydrograph (unit hydrograph) due to a unit (one inch) of excess rainfall applied uniformly to a given watershed over some specified interval, was the tool to be used in runoff predictions.

The basic assumption made in this theory is that a storm runoff derived from the excess rainfall by the use of unit hydrographs is based on linear operation; that is, the composite hydrograph is obtained by adding the simultaneously occuring hydrograph estimates.

To synthesize a unit hydrograph, which can be used to derive a storm hydrograph, a set of coefficients has to be developed based on actual observed runoff data and on physical characteristics of the region.

The following data are needed to use the unit hydrograph method:

1. Basin characteristics
   Drainage area
   Length of main stream
   Slope of the main sewer
   Percent of impervious area

2. Climatological characteristics
   Storm rainfall hyetograph

3. Soil characteristics and Land Use

From a designer's point of view, the unit hydrograph method demands considerably more data, expertise and time than the Rational Method.

A triangular unit hydrograph method developed by the United States Soil Conservation Service is currently being used extensively for flow calculations. The popularity of the method is due to the fact that it can predict complete hydrographs for rural and urban areas using manual or computer methods.

A schematic representation of the unit hydrograph is shown in Figure 14. The necessary parameters to derive the hydrograph are Q_p, unit hydrograph peak in cfs, T_p, time in hours to peak and T_b, time base of unit hydrograph in hours. The unit hydrograph peak, Q_p, resulting from 1 inch of excess rainfall is given by the equation:

$$Q_p = \frac{484}{T_p} A \tag{19}$$

in which

$$T_p = \frac{D}{2} + 0.6\ T_c \tag{20}$$

and

$$T_b = 2.67\ T_p \tag{21}$$

where A = area of subcatchment in square miles
D = excess rainfall period in hours
T_c = time of concentration in hours

The term of 0.6 T_c is an empirical relationship adopted as representative of L, the lag time, which can be defined as the time in hours from the mid-point of the excess rainfall period, D, to the time of peak discharge.

In order to derive the final design flood hydrograph for the sub-catchment under consideration, the developed unit hydrograph must first be converted into incremental triangular hydrographs by multiplying their ordinates by the appropriate direct runoff. The incremental triangular hydrographs so obtained are then summed to form the final design flood hydrograph.

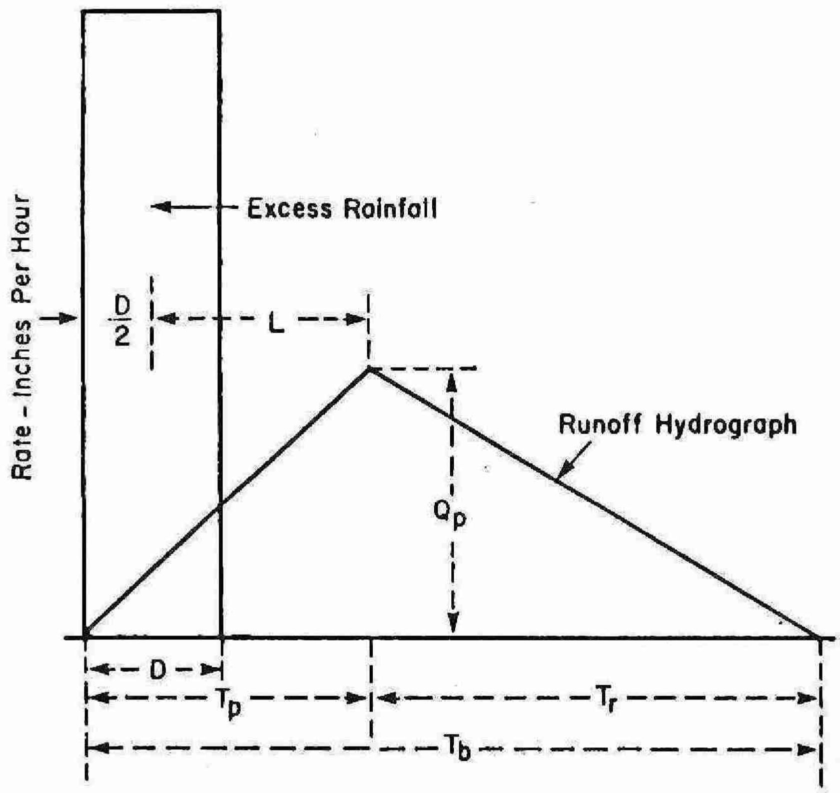


FIGURE 14. SCHEMATIC REPRESENTATION OF UNIT HYDROGRAPH

There are various methods of deriving the time of concentration (T_c). One of the more common and simplest is by using the following equation from California Culverts Practice, California Highways and Public Works, September 1942.

$$T_c = \left(\frac{11.9}{H} L_b^3\right)^{.385} \tag{22}$$

where T_c = the time of concentration in hours
L_b = the length of longest hydraulic path in miles
H = the elevation difference in feet

A more accurate method, however, of determining the time of concentration is by means of stream hydraulics and/or the U.S. Soil Conservation Service method.

4.2.3 REGRESSION MODELS

Regression models seek to relate such factors as rainfall and/or watershed characteristics to an effect such as peak discharge or runoff volume, through statistical correlation. Due to the lack of adequate historical data the use of a regression model in an urban area is minimal.

4.2.4 HYDROLOGIC SIMULATION MODELS

In recent years, the development of mathematical computer simulation programs that model both the quantity and quality of runoff have provided the designer with an invaluable tool.

A hydrologic simulation model in its elementary form is simply a group of mathematical expressions that simulate the rainfall-runoff relationship. The more sophisticated type of model may also reflect infiltration, treatment and storage, and water quality. Many models have been developed which range from crude approximations of the rational method to the solution of many simultaneous differential equations. The single most distinguishing characteristic between simple, intermediate and complex models is the data requirements.

Most hydrologic simulation models require some form of model calibration, using data collected from the study area. This is one of the major problems confronting model applications, particularly when it comes to getting quality measurements which, to be of calibration value, require time related quantity measurements.

Some of the more common hydrologic models associated with the design of minor drainage systems are listed in Table 7. A detailed description of these models can be found in the U.S. Environmental Protection Agency publication entitled "Assessment of Mathematical Models for Storm and Combined Sewer Management" (EPA-600/2-76-175a).

Table 7. LIST OF THE MORE COMMON HYDROLOGIC MODELS

- Battelle Urban Wastewater Management Model
- British Road Research Laboratory Model
- Metropolitan Sanitary District of Greater Chicago Flow Simulation Program
- Chicago Hydrograph Method
- Colorado State University Urban Runoff Modelling
- Corps of Engineers Hydrologic Engineering Center STORM Model
- Dorsch Hydrograph - Volume Method
- Environmental Protection Agency Stormwater Management Model
- Hydrocomp Simulation Program
- Massachusetts Institute of Technology Urban Watershed Model
- Minneapolis-St. Paul Urban Runoff Model
- Municipality of Metropolitan Swattle Computer Augmented Treatment and Disposal System
- SOGREAH Looped Sewer Model
- University of Illinois Storm Sewer System Simulation Model
- University of Massachusetts Urban Runoff Model

- Water Resources Engineers Stormwater Management Model
- Wilsey & Ham Watershed Model
- Illinois State Water Survey Urban Drainage Area Simulator

A number of models shown on Table 7 have come into quite widespread use because of their capabilities, ease of usage, and cost to operate. It is difficult to categorically recommend one model over another with similar characteristics. People have preferences in this regard, and this will continue to be so. In addition, the state-of-the-art is constantly changing and new routines and capabilities are being added.

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